Chapter 4
Outlet Works Seismic Criteria

4-1. General

a. Design and evaluation objectives. Outlet works located in seismic zones 1 through 4 as defined in ER 1110-2-1806 should be seismically evaluated or designed with regard to strength, stability, and serviceability. New intake structures located in seismic zone 0 need not be seismically analyzed unless local site conditions indicate possible response to a nearby seismic event. The criteria set forth in this chapter apply to the structural analysis for earthquake loading of outlet works in accordance with ER 1110-2-1806. The approach accounts for the dynamic characteristics of the tower-water-foundation system, and includes the interaction of the system with the ground motion, gravity forces, and appurtenant structures. The seismic analyses should proceed progressively to produce cost-effective, efficient, and reliable structural configurations and seismic details.

b. Coordination of seismic analysis and design. The design team should normally consist of a structural engineer, a materials engineer, a geotechnical engineer, and an engineering geologist. Such a team should ensure that the seismic analyses, designs, or evaluations are appropriate for the project, and include all aspects of material properties, foundation properties, analysis procedures, and design criteria. The design should be based on historical data, appropriate testing, and engineering judgment. It is essential that the structural engineer be involved in all phases of the material and foundation property assessments, the site seismicity assessment, and the assignment of design ground motions. The basic concepts involved in the seismicity studies must be understood by the structural engineer because he or she must use the results of such studies and therefore must ensure that the information provided is meaningful for structural design or retrofit. In an assessment of the proper level of input, establishing either the level of confidence or the upper and lower bounds will define the sensitive parameters and bracket the results. Such steps should help to ensure that a logical and reasonably conservative design/evaluation of the intake tower could be performed. Assessments should not be based on unnecessary compounding of conservatism because this will lead to an uneconomical design or unnecessary retrofit.

c. Structural stability analysis.

(1) Sliding stability. Preliminary seismic sliding stability can be estimated by the seismic coefficient method. A seismic coefficient equal to two-thirds the peak ground acceleration (PGA) should be used. Shear strength parameters required to perform the sliding analysis should be provided by the engineering geologist or geotechnical engineer. If appropriate testing has not been conducted, an estimated range of values should be provided. The following are minimum required safety factors for seismic sliding analysis:

Operating Basis Earthquake (OBE) = 1.7 for critical structures, and 1.3 for other structures
Maximum Design Earthquake (MDE) = 1.3 for critical structures, and 1.1 for other structures

The computer program CSLIDE may be used to perform the sliding analysis. When rigid body sliding analysis indicates a factor of safety is less than required, a better estimate of safety factor can be made using dynamic analysis or a permanent displacement analysis, if necessary. In the permanent displacement approach the structure is permitted to slide along its base and the accumulated displacement during the ground shaking should be limited to specified allowable values. The analysis method is described in paragraph 2-9*a*(2) of EM 1110-2-6050. The computer program CSLIP may be used.

(2) Rotational stability. For OBE loading conditions, the location of all forces acting on the base of the structure must be such that 75 percent of the base of the structure is in compression. For MDE loading conditions, the resultant location must be within the base. If the resultant location is outside the base, a

dynamic rotational stability analysis should be performed. Rocking occurs when the foundation does not liquefy and

$$S_A > g(b/h) \tag{4-1}$$

where

S_A = spectral acceleration of the first model

g = gravitational acceleration

b = one half of the base width

h = vertical distance from the base to the center of gravity

If rocking occurs, the tower may not be rotationally unstable during the OBE or MDE. Scaling effects, as discussed in Appendix E, show that if S_A is just sufficient to rotate the block through a critical angle α_{cr}, the block will not overturn unless the spectral displacement S_d exceeds one half of the base width. So if a tall slender block is given an initial velocity S_v, which is just sufficient to rock the block to the point of impending static instability, and for small angles (less than 20 degrees).

$$\alpha_{cr} = \frac{S_v}{\sqrt{gr}} \tag{4-2a}$$

and

$$S_d < b \tag{4-2b}$$

where

α_{cr} = the critical angle of rotation from the initial upright position of static equilibrium to the final tipping position of impending rotational instability, radians

S_v = the response spectral velocity for the OBE or MDE ground motion, cm per second

g = gravitational acceleration

r = radial distance from the edge of the base to the center of gravity of the tower, cm

S_d = the response spectral displacement for the OBE or MDE ground motion, cm

The example of a rotational stability of an intake tower is given in Appendix E.

(3) Bearing pressure. Bearing capacity is analyzed according to paragraph 2-6 of EM 1110-2-2200.

4-2. Seismic Evaluation and Design of Tunnels and Cut-and-Cover Conduits

Seismic evaluations of underground structures, where complex structure-foundation interaction is involved, are often based on the performance of similar structures during major earthquakes rather than on the results of numerical analysis. Underground structures are generally less sensitive to seismic effects than surface structures. Information about the performance of underground structures is relatively scarce, largely because of the rarity of failures of underground structures during earthquakes. Underground structures, unless very rigid, deform to accommodate earthquake ground displacements due to traveling seismic waves, ground settlement, lateral spreading, liquefaction-induced deformations, and fault movements. Inertial forces due to ground shaking, as well as any deformational effects due to traveling seismic waves, generally are not significant for underground structures. The ground displacement effects due to lateral spreading of an embankment can be significant, but the effect on cut-and-cover conduits can be difficult to predict. Conduits for outlet works must be subjected to significant ground distortions or permanent displacements due to lateral spreading of the dam embankment or crossing an active fault before they would be expected to experience damage that would lead to failure. A concrete shell will be subjected to compression and extension at points on the exterior and interior of the lining. The exterior extension is of no consequence. In the event that tension cracks appear on the interior surface, they will close again after a fraction of a second. Such cracks do not usually extend through the thickness of the concrete and cannot, in themselves, form a failure mechanism. Information on seismic effects on tunnels can be found in EM 1110-2-2901 along with a simplified method for analyzing tunnels in rock for seismic effects.

4-3. Seismic Evaluation or Design of Intake Structure Bridge

Bridges to intake towers will interact with the tower during an earthquake. The type of interaction will depend on bearing restraint conditions. Most restraint conditions cause a nonlinear response between the bridge and the tower. This nonlinear response is difficult to incorporate into a model that includes both the tower and bridge. Guidance used for modeling the response between bridge superstructures and their supports is provided in Federal Highway Administration (1995). This information is also applicable to modeling bridge-tower interaction. An alternate approach is to assign the bridge mass to the tower, piers, and abutment based on a tributary mass approach. In most cases this approach provides reasonable results since the stiffness of the bridge system usually has little influence on tower behavior. Additional guidance on the seismic analysis methods for bridges and their supports is provided in American Association of State Highway and Transportation Officials (1989).

4-4. Seismic Evaluation or Design of Mechanical and Electrical Equipment

Mechanical and electrical equipment should be seismically protected in accordance with the provision of Tri-Service manual TI 809-04 "Seismic Design for Buildings."

4.5. Seismic Evaluation or Design of Basin Walls (BW)

a. Seismic performance. Case studies of damage to or failure of basin walls induced by ground motions during earthquakes clearly show the need for including appropriate provisions in the design and detailing of walls located in zones of moderate and high seismic activity. Damage to channel walls in the 1971 San Fernando earthquake is described in Clough and Fraguszy (1977). Damage was typically associated with backfill settlement and inward tilting of the walls toward the channel with the center of rotation at the connection between the wall and the channel slab. A few monoliths collapsed into the channel, but most damaged monoliths moved only enough to yield the reinforcement and severely crack the concrete. Severe damage to the walls or a basin induced by lateral backfill pressures may impair capability to release water from the reservoir, and therefore basin walls should be considered a vital feature in the overall seismic design process for outlet works and embankment dams.

b. Progressive analysis. Design of basins depends on the site characteristics, foundation conditions, basin width, and discharge capacity of the outlet works. Basin walls may be a gravity or cantilever structure that may be laterally restrained by the basin slab, or a U-frame structure supported on sound rock, highly overconsolidated soil, or piles. Considering the number of potential design parameters and the complex dynamic nature of the backfill-wall-foundation interaction during earthquakes, it is clear that the seismic design or evaluation of basin walls necessitates many simplifying assumptions. The investigation usually begins with a stability analysis using seismic coefficients and a simplified wedge model. When a rigid body sliding analysis indicates the safety factor is less than required, a permanent displacement analysis should be performed as described in paragraph 4-1*d*(1). In special cases, a dynamic soil-structure interaction (SSI) analysis of the basin walls may be performed using a finite element model. Deciding to proceed to the next level of investigation depends on if the seismic condition is controlling the design or evaluation, and if the relative cost of obtaining information from a more realistic analysis is a better investment than building reserve capacity into the wall system.

c. Basin walls with backfill. The time-history response of walls with backfill is complicated because in addition to the interaction with the foundation rock and water, the wall is also subjected to the dynamic soil pressures induced by ground shaking. Dynamic backfill pressures are related to the relative movement between the soil and the basin wall and the stiffness of the backfill. Behavior of the basin walls may be controlled by a rocking and/or translational response to earthquake shaking. The type of response will correlate to different distributions of backfill pressure acting against the wall. The appropriate method for analyzing the backfill pressure may be categorized according to the expected movement of the backfill and wall during seismic events.

(1) Backfill yields. The relative motion of the wall and backfill material may be sufficiently large to induce a limit or failure state in the soil. This condition may be modeled by the Mononobe-Okabe method (Mononobe and Matuo 1929; Okabe 1924), in which a wedge of soil bounded by the wall and an assumed failure plane are considered to move as a rigid body with the same ground acceleration. The dynamic soil pressures using this approach are described in Chapter 3, Section IV, paragraph 3-26*b* of EM 1110-2-2502.

(2) Backfill does not yield. For sufficiently low intensity ground motions the backfill material may respond within the range of linear elastic deformations. Under this condition the shear strength of the soil is mobilized at a low level so there are no nonlinear deformations in the backfill. The dynamic soil pressures and associated forces in the backfill may be analyzed as an elastic response using Wood's method as described in Ebeling and Morrison (1992).

(3) Backfill partially yields. The intermediate condition in which the backfill soil undergoes limited nonlinear deformations corresponds to the shear strength of the soil being partially mobilized. The dynamic backfill pressures may be idealized as a semi-infinite uniform soil layer using a constant-parameter, single-degree-of-freedom (SDOF) model (Veletsos and Younan 1994) or a frequency-independent, lumped-parameter, multiple-degree-of-freedom (MDOF) system (Wolf 1995). The dynamic pressures for an irregular backfill may be analyzed using a SSI model such as FLUSH (Lysmer et al. 1975). The wall is usually modeled with two-dimensional (2-D) elements. The foundation rock is represented by 2-D plane-strain elements with an appropriate modulus, Poisson's ratio, and unit weight. Transmitting boundaries in the form of dashpots are introduced at the sides of the foundation rock to account for the material nonlinear behavior with depth. The shear modulus and soil damping vary with the level of shearing strain, and this nonlinear behavior is usually approximated by an equivalent linear method. The boundary conditions for the backfill may also be represented by dashpots. Hydrodynamic pressures exerted on the wall are computed using the Westergaard formula.

d. Simplified wedge method. A seismic coefficient method may be used to estimate the backfill and wall inertial forces as described in Chapter 3, Section IV, paragraph 3-26*c* of EM 1110-2-2502. Theoretically, a wall may behave as a rigid body that is fully constrained along its base and sides by the ground, so all parts of the wall may be uniformly affected by accelerations, which are identical to the time-history of the ground motions. Therefore it would be appropriate to use a seismic coefficient equal to the peak ground acceleration for stability analysis of short, stiff walls. However, field and test data show that most walls do not behave as a rigid body, but respond as a deformable body subjected to effective ground motions. Thus the magnitude of the accelerations in a deformable wall may be different from those at the ground surface, depending on the natural period and damping characteristics of the wall and the shaking characteristics of the ground motions. Furthermore the maximum acceleration will affect the wall only for a short interval of time, and the inertia forces will not be equivalent to those of an equal static force that would act for an unlimited time, so the deformations resulting from the maximum acceleration will be smaller. Design or evaluation of basin walls for zero relative displacement under peak ground accelerations is unrealistic, so the seismic stability analysis should be based on a seismic coefficient that recognizes that an acceptably small amount of lateral displacement will likely occur during a major earthquake. Experience has shown that a seismic coefficient equal to two-thirds of the peak ground acceleration is a reasonable estimate for basin walls. For partially yielding backfill, the strength mobilization factor should be equal to the reciprocal of the minimum required sliding safety factor for that load case.

4-6. Seismic Evaluation or Design of Intake Towers

a. Critical classifications for intake towers. According to ER 1110-2-1806, critical features of civil works projects are the engineering structures, natural site conditions, or operating equipment and utilities at high hazard projects whose failure during or immediately following an earthquake could result in loss of life. A critical intake tower is as described based on its capability to lower the reservoir. Damage to or failure of an intake tower located at a high-hazard project may result in a reduced ability to lower the pool following an earthquake. Lowering of the pool may be necessary to relieve pressure head on an embankment dam possibly damaged by earthquake ground motions, or to inspect and repair an embankment dam. In cases where the loss of capacity to lower the pool will result in downstream fatalities, the tower is a critical project feature. If these conditions do not jeopardize lives, the tower is not critical.

b. Design earthquakes and performance requirements. There are two design earthquakes used in accordance with ER 1110-2-1806:

(1) Operational basis earthquake (OBE). The OBE is the level of ground motion for which the structure is expected to remain functional with little or no damage. The OBE is defined as a ground motion having a 50 percent probability of exceedance during the service life of 100 years (a 144-year return period). The associated performance level is the requirement that the structure will function within the elastic range with little or no damage and without interruption of function. In a site-specific study the OBE is determined by a probabilistic seismic hazard analysis (PSHA). The PSHA is described in EM 1110-2-6050.

(2) Maximum design earthquake (MDE). The MDE is the maximum level of ground motion for which the structure is designed or evaluated. The tower may be damaged but retains its integrity. The earthquake performance evaluation of an intake tower for the MDE is based on linear elastic analyses. The evaluation may require postelastic analyses with considerable judgment and interpretation of the results and should be done in consultation with CECW-E.

(a) For critical structures the MDE is set equal to the maximum credible earthquake (MCE). The MCE is defined as the largest earthquake that can reasonably be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806). A site-specific MCE is determined by a deterministic seismic hazard analysis (DSHA). The conditions defining an intake tower as critical are described in *a* above.

(b) For other than critical structures the MDE is selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. Because the purpose of the MDE is to protect against economic losses from damage or loss of service, alternative choices of return period for the MDE may be made on the basis of economic considerations. Ordinarily, the MDE is defined for intake towers as a ground motion having a 10 percent probability of exceedance during the service life of 100 years. This results in a return period of about 1,000 years. For structures a site-specific MDE is determined by a PSHA.

(3) Damping. Damping is a naturally occurring dissipation of energy within the structure. Typical causes of damping are internal friction and hysteretic material behavior. Although damping is mainly a material and structural system-related phenomenon, it is generally incorporated within the response spectra used for analyses. Five percent of critical damping is commonly used for the analysis of intake towers for the MDE and OBE.

(4) Effective stiffness. Postyield response is appropriate when performing seismic design or evaluation for the MDE. Based on Dove (1998), during postyield response the tower will crack at the base and will form a plastic hinge, which will decrease stiffness. The reduced moment of inertia is designated as the effective moment of inertia I_E and is a function of the ratio of the nominal moment capacity M_N to the cracking moment M_{CR}. For intake structures, base the effective moment of inertia on the following relationship:

$$\frac{I_E}{I_g} = 0.8 - 0.9\left[\frac{M_N}{M_{CR}} - 1\right] \tag{4-3}$$

The ratio of I_E/I_G where I_G is the gross moment of inertia (uncracked) should not be greater than 0.8, nor less than 0.35 for walls reinforced with 40 grade steel, nor less than 0.25 for walls reinforced with 60 grade steel. For the OBE the tower should be uncracked at the base; therefore, the effective stiffness is equal to the gross stiffness.

(5) Performance requirements. The performance requirements for intake structures subjected to MDE and OBE demands are provided in *e* below along with the strength requirements for both events.

(6) Earthquake directional components. Earthquake ground motion can be defined for three individual components: a principal horizontal component, a second horizontal component perpendicular to the principal horizontal component, and a vertical component. The three earthquake components should be statistically independent; therefore, the maximum structural responses due to each component will not occur at the same time. It is the designer's responsibility to coordinate with geotechnical engineers on the ground motion components required as described in EM 1110-2-6050. Methods for combining the structural responses from the two horizontal components are described in *e*(1)(d) and (e) below.

c. General design and analysis. The load conditions consist of the usual loads coupled with loads due to a horizontal component of earthquake ground motion. The vertical component of the ground motion has little influence on the overall response of a free-standing tower. Usual loads include the dead weight of the tower, hydrostatic loads due to normal pools, routine operating loads, normal debris loads, and sediment loads. The effects of the design earthquake event should be combined only with other loads that are most likely to occur coincident with normal pool conditions.

(1) Loads. The seismic analysis should account for the dynamic characteristics of the tower-water-foundation system by using the simplified two-mode, lumped-mass method, or by using finite element models. The two-mode, lumped-mass method was developed only for the preliminary design or evaluation of vertical intake structures that are surrounded by water and is not suitable for inclined towers. Finite element models should be used for the final design or evaluation of towers in seismic zones 3 and 4, or in zones 2A or 2B if the seismic load case controls the preliminary design or evaluation.

(2) Analysis. For a vertical tower, the structure may be modeled as a cantilever lumped-mass system of beam elements, and the inertial effects due to the ground motion will induce bending and shear forces in the tower. The earthquake load is modeled by a design response spectrum, or by multiple time-histories for more refined analyses. The response spectrum modal analysis provides only the absolute maximum values of forces, shears, moments, and displacements. The analysis methods and procedures, advantages and disadvantages of response spectrum modal analysis, and procedure for combining modal values are

described in Appendix B. The two-mode approximation and computer solution methods of analysis for a free-standing intake tower is described in Appendix C. A refined method of computing hydrodynamic added mass is described in Appendix D.

d. Torsional effects.

(1) General. Because of the random nature of earthquake ground motions, the direction of seismic loading must be considered. One important consequence of the direction of loading is the potential introduction of torsion. Two categories of towers, regular and irregular, are used to distinguish those towers for which torsional effects are likely to be significant. In deciding whether torsion is significant, the structural engineer should consider all factors such as access bridge effects and local irregularities in the mass or stiffness on a case-by-case basis. A designer should take all reasonable steps to provide symmetry and uniformity without detracting from the functions required for hydraulic purposes.

(2) Regular towers. Regular towers are less subject to damage than irregular towers. Regular towers have a symmetric distribution of mass and stiffness about their principal axes and along the full height. Towers are symmetric in plan if the center of mass is relatively close to the center of rigidity. If these two points are within 10 percent of the tower width when measured perpendicular to the earthquake motion, then the tower is basically symmetric. Towers that are basically symmetrical in plan throughout their height can be designed or evaluated for the bending moments and shears from two-dimensional beam model response spectra or time-history analyses. Earthquakes generate ground motions that occur simultaneously in all directions. For regular towers, the directional uncertainty of the ground motions can be evaluated by performing separate seismic analyses for each principal direction and combining the results as described in *e*(1)(d) below.

(3) Irregular towers. Towers that do not meet the criteria of the preceding paragraph are irregular and should be designed or evaluated for torsion. There are two different approaches for including torsion. The first method uses two-dimensional cantilever beam analyses as performed for the regular towers. On this analysis a separate calculation of torsional effects is superimposed. The torsional moment acting on a cross section is calculated as the sum of all incremental torsional moments above the cross section. An incremental torsional moment is obtained by multiplying the lateral inertial force for a given vertical portion of the tower by its eccentricity from the center of rigidity. The eccentricity is the perpendicular distance from the line of action of the inertial force to the center of rigidity. The resulting summation of the incremental torsional moments above the cross section must then be distributed to the shear-resisting elements according to their relative stiffness and superimposed with the analysis results. This method is common to building evaluations where each wall or lateral force-resisting element of the structure is required to carry both direct shear and torsional shear in accordance with its relative stiffness. A second method for computing torsional effects is to use a three-dimensional structural model to represent the mass and stiffness of the structural system. Various directions of the earthquakes can be investigated to determine the directions that govern the design. In this approach the torsional effects are automatically included in the analysis.

e. Strength and service requirements.

(1) Seismic load cases and combinations.

(a) General. Seismic design for new towers and the evaluation of existing towers must demonstrate that the tower has adequate strength, ductility, and stability to resist the specified earthquake ground motions. Flexural ductility and the benefits of energy dissipation beyond yield should be considered in the design and evaluation process for the MDE, if it can be demonstrated that the tower is not vulnerable to brittle modes of failure. Flexural ductility is indirectly accounted for in the strength equations through the use of a moment reduction factor. Intake structures should have adequate capacity (strength) to resist the MDE demand by allowing for some energy dissipation through inelastic rotation. Intake structures must also have adequate capacity to perform elastically under OBE loading conditions. An example of seismic analysis for the determination of structural capacity is provided in Appendix C.

(b) Limitations. The provisions described herein for the seismic design and evaluation of intake towers are limited to free-standing intake towers with height-to-width aspect ratios of two or more.

(c) Strength requirements. The ultimate strength *U* or capacity of new and existing towers will be determined using the principles and procedures described in EM 1110-2-2104. Capacities are based on ultimate strength, or the nominal strength multiplied by a capacity reduction factor. The capacity reduction factor is 0.9 for bending and 0.85 for shear. Intake tower sections shall have the strength to resist load combinations involving dead load, live load, and earthquake load as prescribed by the following load factor equation:

For the MDE

$$U = D + L + 1.1E / R_M \qquad (4\text{-}4)$$

For the OBE

$$U = 1.4\,(D + L) + 1.5E \qquad (4\text{-}5)$$

where:

U = value of thrusts, shears, or moments due to the effects of dead load, live load, and earthquake

D = internal forces from self-weight

L = internal forces from live loads

E = internal forces from the MDE or OBE. Two orthogonal components must be considered (see the following paragraph).

R_M = moment reduction factor (use $R_M = 1$ for shear and thrust, $R_M = 2$ for moment)

(d) Multicomponent earthquake responses. Intake towers must be capable of resisting maximum earthquake ground motions occurring in any direction. The inertial forces obtained from an analysis generally represent the effects of the principal ground motion component. For a moment, thrust, shear, or force at a particular location, the direction of the earthquake components causing the maximum value needs to be determined. Since an investigation of all possible earthquake directions is difficult, the following alternative methods for estimating peak values should be used.

(e) Circular or rectangular towers. For circular (α = 0.4) or rectangular (α = 0.3) towers, the orthogonal combination method should be used to account for the directional uncertainty of the MDE or OBE earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. This is accomplished by considering the two following combinations:

$$E = \pm[E_X + \propto E_Y] \qquad (4\text{-}6)$$

$$E = \pm[\propto E_X + E_Y] \tag{4-7}$$

where:

E = peak positive or negative values of the forces, shears, moments, or thrust due to the alternating directional effects of the earthquake

E_X = effects resulting from the X-component of ground motion occurring in the direction of the major principal tower axis

E_Y = effects resulting from the Y-component of ground motion occurring in the direction of the minor principal tower axis.

(f) Unsymmetrical or inclined towers. The square root of sum of square (SRSS) method should be used for a preliminary evaluation of the multidirectional earthquake effects on towers that are irregular in plan and elevation. The force, shear, moment, or thrust at a particular location can be estimated from any set of orthogonal analyses by a SRSS combination.

$$E = \sqrt{E_X^2 + E_Y^2} \tag{4-8}$$

(2) Ductility.

(a) Inelastic behavior. Ductility is an indicator of inelastic deformation. Research performed by Dove (1998) shows that a reinforced concrete tower should have a displacement at failure that is equal to 150 to 350 percent of the displacement at yield. Based on a corresponding bending displacement ductility of 1.5 to 3.5, a tower should be designed or evaluated for seismic moments that are 70 to 40 percent, respectively, of the elastic moments obtained from a response spectrum or time-history analysis. Some load transfer mechanisms in reinforced concrete are brittle, and the strength may degrade after repeated cycles of loading beyond the yield level. The brittle failure modes are due to fracture of the reinforcement, an anchorage failure of the reinforcement, a splice failure (bond) of the reinforcement, a shear (diagonal tension) failure, a compressive spalling failure, and a sliding shear failure.

(b) Moment reduction factor (R_M). The moment reduction factor is the ratio of the elastic response moment to the applied seismic moment. A moment reduction factor of two should be used only when brittle failure modes cannot occur, and the structural engineer should design and detail the structure so that all the brittle failure modes are precluded. Properly detailing the reinforcement is important to assure adequate performance and to preserve the structural integrity of the tower during a seismic event. Adequate detailing of reinforcement should be provided in accordance with the provisions in (3) to (8) below. An example is in Appendix C.

(c) Confinement requirements. Some towers may require special confinement reinforcement and boundary elements. A moment reduction factor greater than two should be used only if appropriate measures are taken to confine the concrete in regions subjected to large compressive strains, and to improve the cyclic performance of splices and anchorages by providing heavy confinement reinforcement in plastic hinge regions. Plastic hinges or inelastic zones occur in regions with large moments. The confining reinforcement keeps the concrete from spalling and allows the tower to redistribute loads after cracking. Typical locations occur at the base of conduits and other openings, or at discontinuities due to abrupt changes in structural geometry. Corners should have diagonal reinforcement to control anticipated cracks. Evaluation of postelastic and moment reduction factors greater than two should be performed in consultation with and approved by CECW-EW.

(3) Fracture of reinforcement requirements.

(a) General. The tensile reinforcement can fracture suddenly if the reinforcement is less than 1 percent and cause a brittle failure. To prevent a brittle failure mechanism, the nominal moment capacity should equal or exceed the uncracked moment capacity by at least 20 percent. The cracking moment can be calculated using Equation 4-9:

$$M_{cr} = \left(\frac{I_g}{C}\right)\left(\frac{P}{A_G} + f_r\right) \tag{4-9}$$

where

C = the distance from the neutral axis to the extreme fiber

P = axial load on the tower (positive for tension)

A_g = gross section area (uncracked)

f_r = modulus of rupture, $0.62\sqrt{f_c'}$ (MPa units), 7.5 $\sqrt{f_c'}$ (psi units) where f_c' is the concrete compression strength

(b) Existing towers. A force-based or a displacement-based approach may be used to evaluate the minimum reinforcement requirement if the nominal moment capacity is less than 120 percent of the cracking moment.

(c) New towers. For all seismic designs, sufficient reinforcing steel should be provided to assure that the nominal moment capacity equals or exceeds 120 percent of the cracking moment (Equation 4-10):

$$M_N \geq 1.2 M_{cr} \tag{4-10}$$

(4) Anchorage failure mode.

(a) Existing towers. The flexural strength of the intake tower will deteriorate during a major earthquake if the vertical reinforcement is not adequately anchored. For straight bars, the anchorage lengths should be greater than required by Equation 4-11:

$$l_a = \frac{2.626 k_s d_b}{\sqrt{f_c'}\left(1 + 2.5\frac{c}{d_b}\right)} \quad \text{(KPa units)} \tag{4-11}$$

$$l_a = \frac{k_s d_b}{\sqrt{f_c'}\left(1 + 2.5\frac{2}{d_b}\right)} \quad \text{(psi units)}$$

where

l_a = minimum required effective anchorage length of longitudinal reinforcement

k_s = a constant for reinforcing steel with a yield stress of f_y

$$k_s = \frac{\left(f_y - 76845\right)}{33.1} \qquad \text{(KPa units)}$$

$$k_s = \frac{\left(f_y \text{ -}11000\right)}{4.8} \qquad \text{(psi units)}$$

f_y = yield stress of the longitudinal reinforcement

d_b = nominal bar diameter

c = the lesser of the clear cover over the bars, or half the clear spacing between adjacent bars, in.

The term $2.5(c/d_b)$ introduces the effect of clear cover to the calculation of development length. The value of (c/d_b) shall not be greater than 2.5. For 90-degree standard hooks, the anchorage provided should be greater than that required by Equation 4-12. For anchorage with 90-degree standard hooks, the effective anchorage length in inches is

$$l_a = 1,200 d_b \frac{2.626 f_y}{60,000\sqrt{f_c'}} \qquad \text{(KPa units)}$$

(4-12)

$$l_a = 1,200 d_b \frac{f_y}{60,000\sqrt{f_c'}} \qquad \text{(psi units)}$$

(b) New towers. The required effective anchorage length of longitudinal reinforcement should be equal to or greater than that provided by Equations 4-11 and 4-12. The minimum anchorage length, however, should not be less than 30 bar diameters for straight anchorage or less than 15 bar diameters for hooked anchorages.

(5) Splice failure of bending reinforcement.

(a) Existing towers. Splices in the flexural steel, if located in a plastic hinge region, may undergo strength deterioration during a major earthquake. When compressive strains in the concrete are less than 0.002 in./in., the splices will perform satisfactorily without transverse confinement steel. When concrete compressive strains exceed 0.002 in./in., transverse confinement steel should be provided at splice locations. The minimum area of transverse reinforcement A_{tr} necessary to prevent bond deterioration is:

$$A_{tr} = \frac{s f_y}{l_s f_{yt}} A_b \qquad (4\text{-}13)$$

where

A_{tr} = minimum area of transverse reinforcement

s = average spacing of transverse reinforcement over the splice length

l_s = splice length

f_{yt} = yield stress of the transverse reinforcement

A_b = area of the spliced bar

(b) New towers. Deterioration of bond and splice strengths of reinforcing bars is a critical potential failure mode for reinforced concrete structures. Transverse reinforcement provides the best protection against splice strength degradation. Perimeter transverse reinforcing steel in the amount equal to or greater than that indicated by Equation 4-13 should be provided at all splice locations where concrete compressive strains are expected to exceed 0.002 in./in. Transverse steel provided to resist shear and bending forces induced by hydrostatic or gravity loads may also be considered as effective transverse confinement steel and used to prevent splice failures. Perimeter transverse confinement reinforcement using smaller bars at close spacings is better than that obtained using larger bars at wide spacing.

(c) Splice length requirements. Splice performance will be greatly improved if splices are located away from potential plastic hinge regions and if lap splice locations are staggered (i.e., no more than half of the bars spliced at any horizontal plane). The lap splice length required for new and existing towers should not be less than 154 $d_b \div \sqrt{f_c'}$ MPa units (1,860 $d_b \div \sqrt{f_c'}$ psi units). Existing towers with lesser splice lengths may perform acceptably if splices are staggered. Lesser splice lengths should be evaluated according to their location and seismic demand.

(6) Shear (diagonal tension) failure. For existing and new towers, the capacity of the intake tower in shear shall be equal to or greater than the lesser of the full elastic demand placed on the tower by the design earthquake, or the shear corresponding to 1.5 times the shear associated with the nominal flexural strength of the tower. The capacity of the concrete in shear and the shear resistance available from the transverse reinforcing after deductions for other live load effects may be considered. The shear capacity of the tower includes contributions from both the concrete and the shear reinforcement. The total ultimate shear strength V_U is

$$V_U = \theta\,(V_C + V_S) = 0.85\,(V_C + V_S) \tag{4-14}$$

where V_C is the contribution from the concrete:

$$V_c = 2\left[K + \frac{P}{13.8A_g}\right]0.083\sqrt{f'_{CA} \cdot A_E} \qquad \text{(KPa units)}$$

(4-15)

$$V_c = 2\left[K + \frac{P}{2{,}000A_g}\right]\sqrt{f'_{CA} \cdot A_E} \qquad \text{(psi units)}$$

where

$K = 1$ for $R_M = 1$; $K = 0.5$ for $R_M = 2$

P = axial load on the section

f'_{CA} = actual concrete compression strength (typically $f'_{CA} \geq 1.5 f'_C$)

$A_E = 0.8\ A_{gross}$ (A_{gross} = actual cross-section area)

and V_S is the contribution from the shear reinforcement as defined for circular towers as equal to:

(a) For circular towers:

$$V_s = \frac{\pi A_h (f_y)(0.8d)}{2s} \tag{4-16}$$

where

A_h = horizontal reinforcement cross-section area

d = outside diameter of the tower

f_y = stress in the transverse reinforcing due to unfactored hydrostatic loading and other non-earthquake loads

s = spacing of reinforcement

(b) For rectangular towers,

$$V_s = \frac{A_h (f_y)(0.8d)}{s} \tag{4-17}$$

where d is the tower dimension in the direction of the seismic shear force. Since this method uses an interaction equation to evaluate biaxial shear (see Appendix C), the earthquake forces must be in the directions of the principal axes.

(7) Compressive spalling failure. In existing and new towers, excessive compressive strains can cause spalling of the concrete cover and rapid degradation of the transverse confining reinforcement, which can lead to splice failures and buckling of the longitudinal reinforcing steel. However, the chance of compressive spalling failures is small because the compressive stresses and the reinforcing steel percentages are low in intake towers. Compressive spalling failures will not occur at ultimate load conditions if the concrete compressive strains are less than 0.4 percent, or if the location of the neutral axis is less than 15 percent of the effective depth to the centroid of the reinforcement. The latter condition can be expressed as

$$\frac{c}{d} \leq 0.15 \tag{4-18}$$

where

c = distance from extreme compression fiber to the neutral axis, in.

d = distance from extreme compression fiber to the centroid of tension reinforcement, in.

(8) Sliding shear failure investigation. In existing and new towers, the potential for sliding along a horizontal crack at all possible failure planes within the structure should be evaluated. The nominal sliding

shear strength can be used to determine if a sliding shear failure is possible. The nominal shear strength can be determined using Equation 4-19:

$$V_{SL} = P + 0.25 f_y A_{VF} \tag{4-19}$$

(Assuming coefficient of friction $\mu = 1$)

where

V_{SL} = nominal sliding shear strength

P = axial load on section

f_y = yield strength of reinforcement

A_{VF} = area of shear friction reinforcement (area of vertical reinforcement crossing the horizontal failure plane)

The sliding shear resistance in KN (kips) calculated by use of Equation 4-19 shall not be greater than

$$0.0008\sqrt{f_c'}\,A_g \text{ MPa} \quad (0.01\sqrt{f_c'}\,A_g \text{ psi})$$

where A_g is the gross area of concrete normal to the horizontal failure plane. It should be noted that sliding shear does not necessarily constitute a failure condition, since the permanent displacements that occur during sliding may not be sufficient to cause structural instability.

(9) Displacement-based analysis of existing rectangular and circular towers.

(a) Tower performance. Many existing intake towers are lightly reinforced structures (steel percentage less than 0.50 percent) that may have a nominal moment capacity less than 120 percent of the cracking moment. If the nominal moment capacity exceeds the cracking moment, then the tower will be able to form a plastic hinge at critical points of structural damage similar to the behavior of reinforced concrete bridge piers or building columns. Towers that do not meet this minimum requirement will have plastic hinge lengths that are much less than those in buildings or bridges, and these towers are more likely to fail by fracturing the longitudinal reinforcement. Recent analytical and experimental research that addressed the nonlinear seismic response and ductility of lightly reinforced towers has shown that many towers will have limited ductility corresponding to a highly localized failure of the reinforcement within a single crack (Dove 1998). The critical crack may form at the base of the tower or at other abrupt changes in structural stiffness. Typical rectangular or circular towers will have small plastic hinge lengths with a limited displacement capacity.

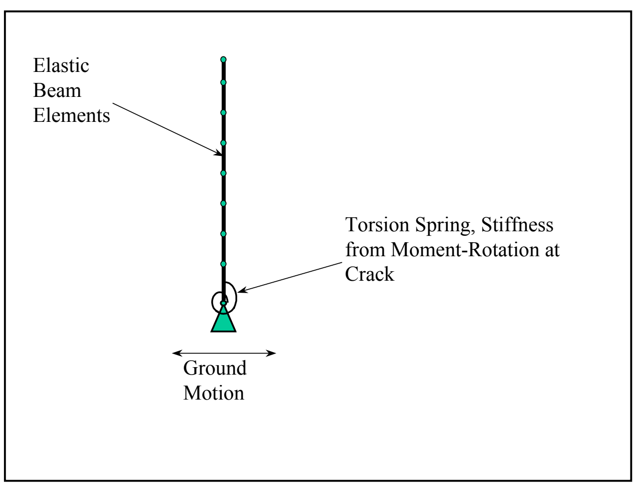

Figure 4-1. Tower model of displacement demand

(b) Displacement demand. A displacement-based dynamic analysis may be used to evaluate this localized failure mode, and to determine if the ultimate displacement capacity δ_U at the top of the tower (as shown in Figure 4-1) exceeds the displacement demand δ_D for the MDE. This technique is a modified response spectrum analysis that considers the earthquake-induced displacements of a tower, and uses effective stiffness properties (*b*(4) above) to account for the shift in the fundamental frequencies of the tower due to formation of plastic regions. The displacement demand is the maximum deflection calculated at the top of the tower from a response spectrum analysis using a linear spring stiffness, the beam element properties, and the added mass due to surrounding or contained water. The tower is modeled as a cantilever beam that is supported at the base on a rotational spring (Figure 4-1). The beam represents the elastic response of the tower above the crack, and the spring represents the nonlinear response of the cracked region. The nonlinear rotational stiffness of the spring may be estimated by using an appropriate reinforced concrete section analysis computer program to calculate the moment-curvature relationship for the cross-section of the tower, and then multiplying the curvature values by an appropriate plastic hinge length to plot the corresponding moment-rotation relationship. The moment-rotation relationship represents the stiffness of the rotational spring, and this relationship is often strongly bilinear. The following simplification is required to approximate the spring stiffness as a linear relationship in the elastic response spectrum analysis. First, calculate the area under the bilinear moment-rotation curve from the origin to the value of the maximum allowable rotation, and then generate a fictitious linear, moment-rotation curve that has the same area and maximum rotation. The fictitious spring stiffness should have the same total energy and maximum rotation as the bilinear curve, but the rotational stiffness and the frequency of spring response will lie between the real elastic and inelastic values.

(c) Displacement capacity. The displacement capacity at the top of the tower is related to the height, the length of the plastic hinge, and the fracture strain capacity (approximately 0.05 in./in.) of the reinforcement. The displacement capacity may be modeled as shown in Figure 4-2 and discussed below.

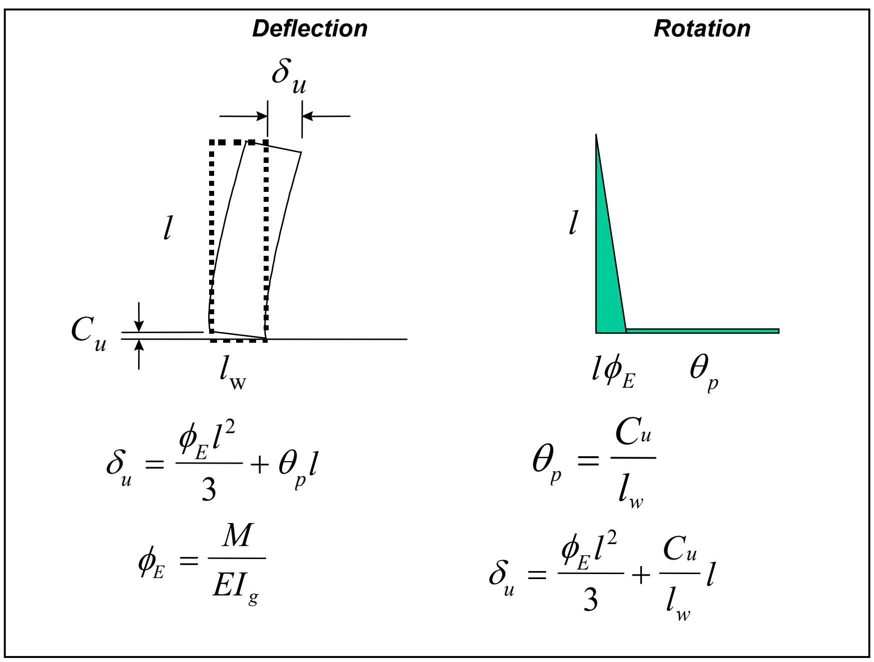

Figure 4-2. Tower model of displacement capacity

where

δ_u = the ultimate displacement capacity

ϕ_E = the elastic curvature at cracking (at the base of the tower)

l = the height of the tower above the crack

θ_p = the plastic rotation at failure

M = the moment at the base of the tower

C_u = the ultimate crack width at failure

l_w = the depth of the section

The ultimate lateral deflection is modeled as the sum of an elastic response of the tower above the cracked section and a rigid body rotation of the tower as the crack opens at the base. It is conservative to assume the tower rotates about a neutral axis of the cracked section that is coincident with the edge of the tower. Hence, the lateral rigid body deflection at the top of the tower varies directly with the crack width, and its maximum value varies as a ratio of the tower height to the tower width, times the ultimate crack width. The ultimate crack width at failure C_u is a function of the ultimate strain in the reinforcement and the strain penetration. Recent experiments have modeled the cyclic response of single bars for different concrete strengths, bar sizes, and bar strengths. Statistical analyses of these experiments indicate that for a single crack response, the crack widths are reliably predicted by the ultimate strain capacity of the bar and the bar diameter. An appropriate value of the ultimate crack width may be estimated using the following empirical equation:

$$C_u = 0.12 + 2.47\varepsilon_u + 0.312d_b \tag{4-20}$$

where

ε_u = ultimate strain at failure of the bar as measured over a standard 20.32-cm (8-in.) gage length

d_b = diameter of the reinforcing, cm

(d) Tower example. A displacement-based evaluation of the tower in Appendix C is based on a failure mechanism that may be characterized by a single crack at the base of the tower. The analysis is summarized below.

(1) Compute the moment-curvature relationship for the bottom section of the tower about the weak and strong axes, and include the deadweight of the tower. For an 18 percent ultimate strain, the ultimate crack width and the strain penetration length L_s may be estimated as 1.65 cm (0.65 in.) and approximately 10.0 cm (4.0 in.), respectively, from the previous empirical equation and the following equation:

$$L_s = C_u / \varepsilon_u \tag{4-21}$$

(2) Multiply the moment-curvature values by this strain penetration length to obtain the moment-rotation relationship (Figure 4-3). The values of the linear spring stiffness for bending about the weak and strong axes are estimated based on having equal total areas under the actual and fictitious moment-rotation curves. The rotational spring constants for the weak and strong axes are 2.107E+12 N-m/Radian and 3.561E+12 N-m/Radian, respectively.

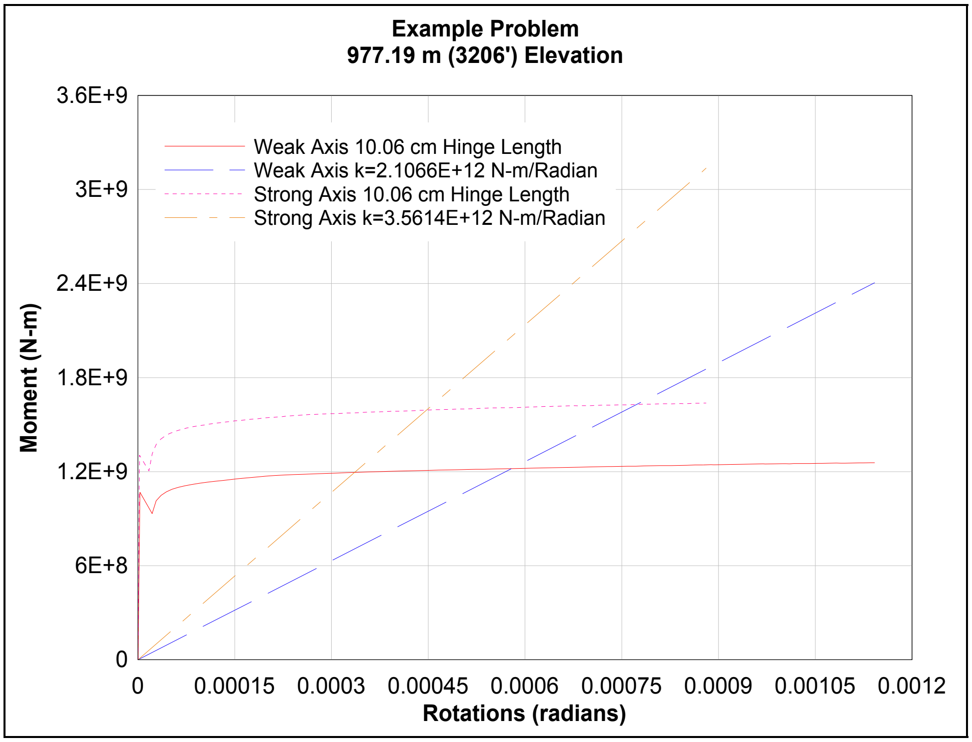

Figure 4-3. Moment-rotation relationship for the tower base

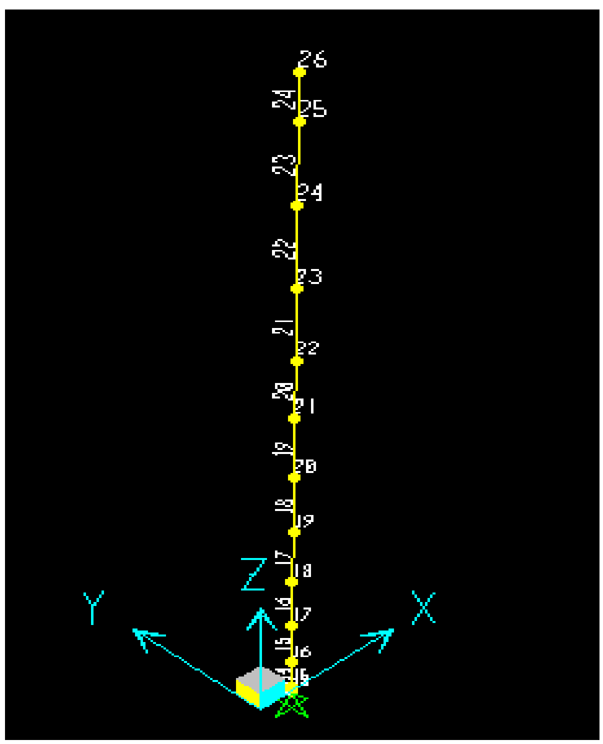

Figure 4-4. SAP 2000 intake tower model

(3) Perform a structural analysis for the MDE response spectrum with 5 percent damping (as shown in Figure C-3) using commercial software such as SAP 2000. This is similar to the analysis presented in Section C-8 except for the hinge supports and rotational springs provided at the base of the tower to model the properties of the cracked section (Figure 4-4). The same section properties and masses were used (as shown in Figures C-1 and C-5) to model the tower and water. The sectional properties of the tower were uniformly distributed, and the added masses $\left(m_a^o + m_a^i\right)$ were lumped at nodes only (as shown in Tables C-3 and C-4). The deflections calculated at the top of the tower are 9.6 cm (3.8 in.) and 10.1 cm (4.0 in.) for the strong and weak axes, respectively.

(4) The elastic curvature ϕ_E at the base is read from the moment-curvature relationship for the section. The ultimate base rotation and the ultimate displacement capacity are calculated using the following equations:

$$\theta_p = C_u / l_w \tag{4-22}$$

$$\delta_u = \frac{\phi_e l^2}{3} + \theta_p l \tag{4-23}$$

(5) The deflection capacities at the top of the tower are 10.7 cm (4.2 in.) and 14.2 cm (5.6 in.) about the strong and weak axes, respectively. Therefore the tower satisfies the performance requirements for the MDE.

4-7. Special Guidance for Inclined Intake Towers

a. Inclined and vertical towers supported along one side by the rock abutment will experience complicated bending, shear, and torsion under earthquake ground shaking. The ground motions excite the tower not only from the base but also along the entire abutment support. The three-dimensional geometry of supported towers combined with composite seismic input requires a three-dimensional finite element modeling of the tower for earthquake response analysis. Generally, supported towers should be discretized using a combination of three-dimensional solid and shell elements, as appropriate, or simply by three-dimensional solid elements. However, some components of the tower such as the trashrack can be modeled more appropriately using beam elements. The interaction with the foundation and abutment may be considered by including a portion of the foundation and abutment regions as part of the tower model. The foundation and abutment models should be developed to have dimensions comparable to those of the tower section. Competent rock sites may adequately be represented by a massless foundation model while foundation soil models are usually represented by equivalent springs, by a combination of springs and dashpots, or by finite element soil models. More detailed guidance on foundation-structure interaction is covered in EM 1110-2-6050 and EM 1110-2-6051.

b. The effect of hydrodynamic pressures on supported towers is represented by the equivalent added-mass concept. The added hydrodynamic mass, however, depends not only on the geometry of the tower but also on topography of the surrounding abutment-foundation region. If the effect of topography on the added mass is judged significant, the added mass should be computed using the finite element or boundary element procedures as discussed in EM 1110-2-6051.

c. The dynamic response of a supported tower is a complex combination of biaxial bending and torsion depending on the tower-abutment-foundation and tower-water interaction. This condition is best treated by three-dimensional response-spectrum (EM 1110-2-6050) and time-history analyses (EM 1110-2-6051). The analysis may start with the response-spectrum modal superposition followed by time-history procedure. The seismic input consists of three components of ground motions in the form of response spectra or acceleration time-histories applied at the fixed exterior nodes of the foundation model.

A response-spectrum analysis provides only maximum stresses and section forces. A time-history analysis provides both the maximum magnitudes and distributions of stresses and section forces at any time during a seismic event. In case of severe seismic events, both magnitudes and time variation of stresses and forces may be required to design or evaluate the structure and the anchorage system.

4-8. Remedial Strengthening of Existing Towers

The seismic analysis indicates an existing intake tower is in need of a seismic retrofit, or when nonlinear, ineleastic seismic analysis procedures are necessary to investigate performance of the structure. The need for remedial strengthening will be determined on a case-by-case basis after suitable ductility evaluation has been performed. Any evaluations of existing intake towers that indicate the need for remedial strengthening shall be done in consultations with CECW-EW.